2. INSPECTION AND CLASSIFICATION OF DAMAGE

2.1 Introduction

This chapter defines a uniform system for classification and reporting of damage to steel moment-frame structures that have been subjected to strong earthquake ground shaking.

Structural damage observed in steel moment-frame buildings following strong ground shaking can include yielding, buckling and fracturing of the steel framing elements (beams and columns) and their connections, as well as permanent lateral drift. Damaged elements can include girders, columns, column panel zones (including girder flange continuity plates and column web doubler plates), the welds of the beam to column flanges, the shear tabs which connect the girder webs to column flanges, column splices and base plates. Figure 2-1 illustrates the location of these elements.

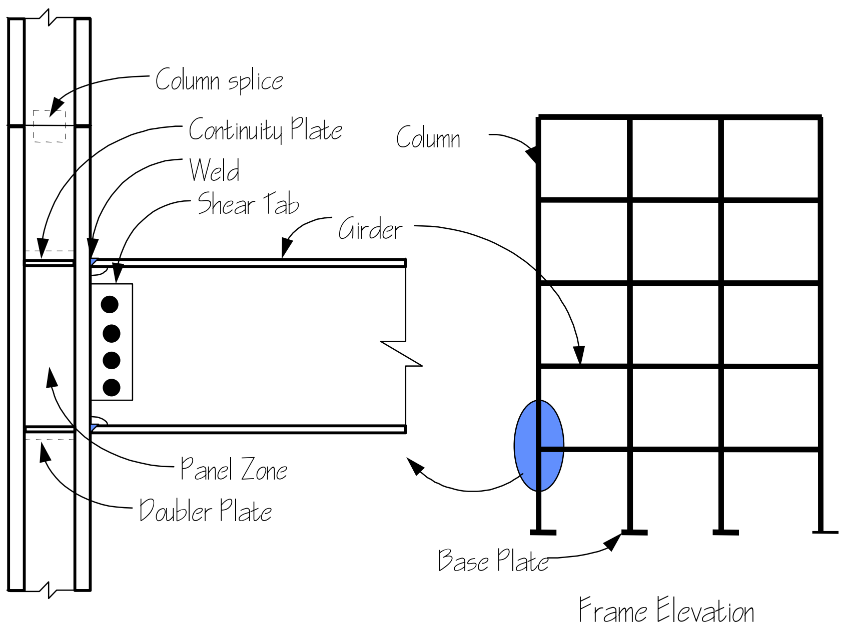

Figure 2-1 Elements of Welded Steel Moment Frame

2.2 Damage Types

Damage to framing elements of steel moment-frame buildings may be categorized as belonging to the weld (W), girder (G), column (C), panel zone (P) or shear tab (S) categories. This section defines a uniform system for classification and reporting of damage to elements of steel moment-frame structures that is utilized throughout these *Recommended Criteria*. The damage types indicated below are not mutually exclusive. A given girder-column connection, for example, may exhibit several different types of damage. In addition to the individual element damage types, a damaged steel moment-frame may also exhibit global effects, such as permanent interstory drifts.

Following a detailed postearthquake inspection, classification of the damage found, as to its type and degree of severity, is the first step in performing an assessment of the condition and safety of a damaged steel moment-frame structure. In a level 1 evaluation, conducted in accordance with Chapter 4 of these *Recommended Criteria,* the classifications of this section are used for the assignment of damage indices. These damage indices are statistically combined and extrapolated to provide an indication of the severity of damage to a structure's lateral force resisting system and are used as a basis for selecting building repair strategies. For a level 2 evaluation, conducted in accordance with Chapter 5 of these *Recommended Criteria,* these damage classifications are keyed to specific modeling recommendations for analysis of damaged buildings to determine their response to likely ground shaking in the immediate postearthquake period. Chapter 6 addresses specific techniques and design criteria recommended for the repair and modification of the different types of damage, keyed to these same damage classifications.

Commentary: The damage types contained in this chapter are based on a system first defined in a statistical study of damage reported in NISTR-5625 (Youssef et al., 1995). The original classes contained in that study have been expanded somewhat to include some conditions not previously identified.

2.2.1 Girder Damage

Girder damage may consist of yielding, buckling or fracturing of the flanges of girders at or near the girder-column connection. Seven separate types are defined in Table 2-1. Figure 2-2 illustrates these various types of damage. See Section 2.2.3 and 2.2.4 for damage to adjacent welds and shear tabs, respectively.

Commentary: Minor yielding of girder flanges (type G2) is the least significant type of girder damage. It is often difficult to detect and may be exhibited only by local flaking of mill scale and the formation of characteristic visible lines in the material, running across the flange. Removal of finishes, by scraping, may often obscure the detection of this type of damage. Girder flange yielding, without local buckling or fracture, results in negligible degradation of structural strength and typically need not be repaired.

Table 2-1 Types of Girder Damage

Type	Description
G1	Buckled flange (top or bottom)
G2	Yielded flange (top or bottom)
G3	Flange fracture in Heat Affected Zone (top or bottom)
G4	Flange fracture outside Heat Affected Zone (top or bottom)
G5	Not used
G6	Yielding or buckling of web
G7	Fracture of web
G8	Lateral torsion buckling of section

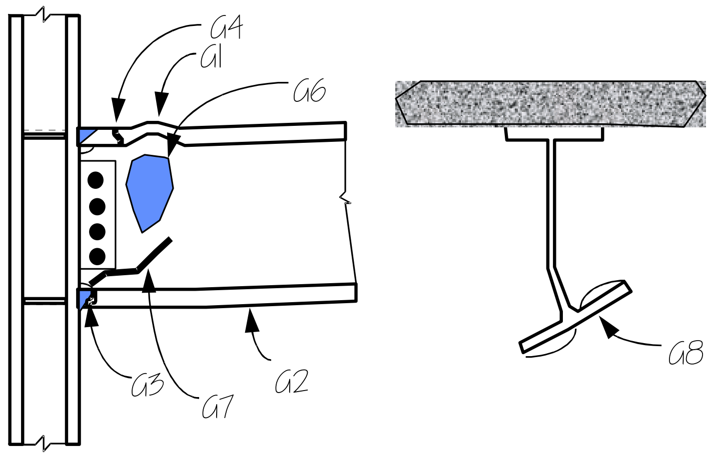

Figure 2-2 Types of Girder Damage

Girder flange buckling (type G1) can result in a significant loss of girder plastic strength, particularly when accompanied by girder web buckling (type G6). For compact sections, this strength loss occurs gradually, and increases with the number of inelastic cycles and the extent of the inelastic excursion. Following the initial onset of buckling, additional buckling will often occur at lower load levels and result in further reductions in strength, compared to previous cycles. The localized secondary stresses which occur in the girder flanges due to the buckling can result in initiation of flange fracture damage (G4) if the frame is subjected to a large number of cycles. Such fractures typically progress slowly over repeated cycles, and grow in a ductile manner. Once this type of damage initiates, the girder flange will begin to lose tensile capacity under continued or reversed loading, although it may retain some capacity in compression. Visually evident girder flange buckling should be repaired.

In structures with weld material with low notch-toughnesss, girder flange cracking within the Heat Affected Zone (HAZ) (type G3) can occur as an extension of brittle fractures that initiate in the weld root. This is particularly likely to occur at connections in which improper welding procedures were followed, resulting in a brittle HAZ. However, these fractures can also occur in connections with welded joints made with notch-tough weld metal and following appropriate procedures, as a result of low-cycle fatigue, exacerbated by the very high strain demands that occur at the toe of the weld access hole, in unreinforced beam-column connections. Like the visually similar type G4 damage, which can also result from low cycle fatigue conditions at the toe of the weld access hole, it results in a complete loss of flange tensile capacity, and consequently, significant reduction in the contribution to frame lateral strength and stiffness from the connection.

In the 1994 Northridge earthquake girder damage was most commonly detected at the bottom flanges, although some instances of top flange failure were also reported. There are several reasons for this. First, the composite action *induced by the presence of a floor slab at the girder top flange tends to shift the neutral axis of the beam towards the top flange. This results in larger tensile deformation demands on the bottom flange than on the top. In addition, the presence of the slab tends to reduce the chance of local buckling of the top flange. The bottom flange being less restrained can experience buckling relatively easily. Finally, much of the damage found in girders initiates as a result of defects at the root of the beam flange to column flange weld. Due to its position, the weld of the bottom beam flange to column flange is more difficult to make than that at the top flange, and therefore, is more likely to have defects that can initiate such damage.*

2.2.2 Column Flange Damage

Seven types of column flange damage are defined in Table 2-2 and illustrated in Figure 2-3. Column flange damage typically results in degradation of a structure's gravity-load-carrying strength as well as lateral-load resistance. For related damage to column panel zones, refer to Section 2.2.5.

Table 2-2 Types of Column Damage

Type	Description
C1	Minor column flange surface crack
C2	Flange tear-out or divot
C3	Full or partial flange crack outside Heat Affected Zone
C4	Full or partial flange crack in Heat Affected Zone
C5	Lamellar flange tearing
C6	Buckled flange
C7	Column splice failure

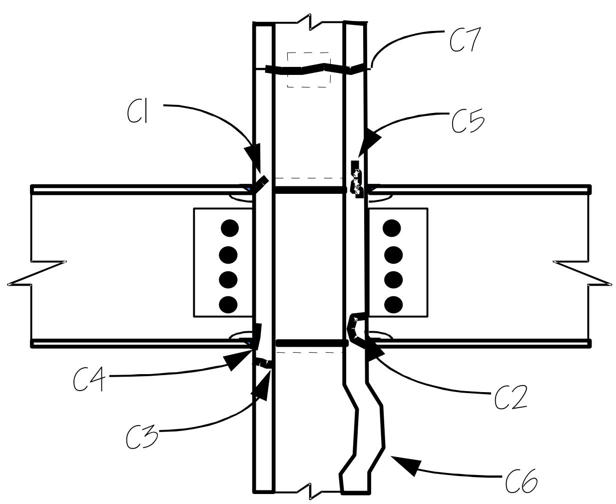

Figure 2-3 Types of Column Damage

Commentary: Column flange damage includes types C1 through C7. Type C1 damage consists of a small crack at the surface of the column flange and extending into its thickness, typically at the location of the adjoining girder *flange. C1 damage does not go through the thickness of the column flange and can often be detected only by nondestructive testing (NDT). Type C2 damage is an extension of type C1, in which a curved failure surface extends from an initiation point, usually at the root of the girder-to-column-flange weld, and extends longitudinally into the column flange. In some cases this failure surface may emerge on the same face of the column flange as the one where it initiated. When this occurs, a characteristic "nugget" or "divot" can be withdrawn from the flange. Types C3 and C4 fractures extend through the thickness of the column flange and may extend into the panel zone. Type C5 damage is characterized by a step-shaped failure surface within the thickness of the column flange and aligned parallel to it. This damage is often detectable only with the use of nondestructive testing.*

Type C1 damage does not result in an immediate large strength loss in the column; however, such small fractures can easily progress into more serious types of damage if subjected to additional large tensile loading by aftershocks or future earthquakes. Type C2 damage may result in both a loss of effective attachment of the girder flange to the column for tensile demands and could cause a significant reduction in available column flange area for resistance of axial and flexural demands. Type C3 and C4 damage result in a loss of column flange tensile capacity and under additional loading can progress into other types of damage.

Type C5 damage may occur as a result of non-metallic inclusions within the column flange. The potential for this type of fracture under conditions of high restraint and large through-thickness tensile demands, such as the residual stresses induced by welding, has been known for a number of years, and is termed lamellar tearing. There is no evidence that lamellar tearing actually occurred in buildings as a result of earthquake ground shaking and it is currently thought that when type C5 damage did occur, it was an extension of fracturing that initiated in the weld root. This damage has sometimes been identified as a potential contributing mechanism for type C2 column flange through-thickness failures. Note that in many cases, type C2 damage may be practically indistinguishable from type W3 fractures (see Section 2.2.3). The primary difference is that in type W3, the fracture surface generally remains with in the heat affected zone of the column flange material while in C2 damage, the fracture surface progresses deeper into the column flange material.

Type C6 damage consists of local buckling of the column flange, adjacent to the beam-column connection. While such damage was not actually observed in buildings following the 1994 Northridge earthquake, it can be anticipated at locations where plastic hinges form in the columns. Buckling of beam flanges has been observed in the laboratory at interstory drift demands in excess of 0.02 radians. Column sections are usually more compact than beams and therefore, *are less prone to local buckling. Type C6 damage may occur, however, in buildings with strong-beam-weak-column systems and at the bases of columns in any building when very large interstory drifts have occurred.*

Type C7 damage, fracturing of welded column splices, also was not observed following the Northridge earthquake. However, the partial joint penetration groove welds commonly used in these splices are very susceptible to fracture when subjected to large tensile loads. Large tensile loads can occur on a column splice as a result of global overturning effects, or as a result of large flexural demands in the column.

As a result of the potential safety consequences of complete column failure, all column damage should be considered as significant and repaired expeditiously.

2.2.3 Weld Damage

Three types of weld damage are defined in Table 2-3 and illustrated in Figure 2-4. All apply to the complete joint penetration welds between the girder flanges and the column flanges.

Table 2-3 Types of Weld Damage, Defects and Discontinuities

Type	Description
W1, W1a, W1b	Not Used (see commentary)
W2	Crack through weld metal thickness
W3	Fracture at column interface
W4	Fracture at girder flange interface
W5	Not Used (see commentary)

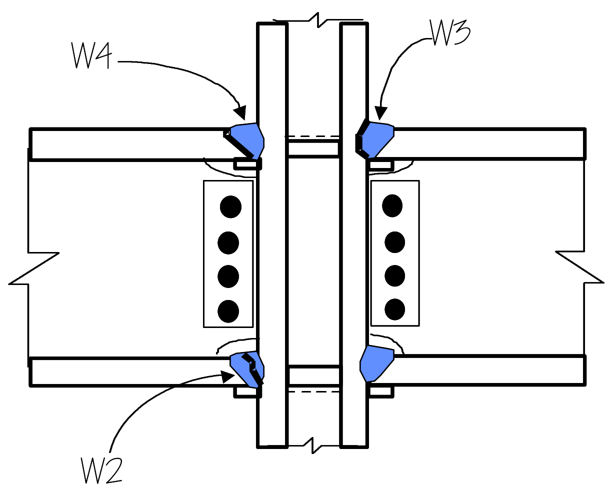

Figure 2-4 Types of Weld Damage

Commentary: In addition to the W2, W3, and W4 types of damage indicated in Table 2-3 and Figure 2-4, the damage classification system presented in FEMA-267 included conditions at the root of the complete joint penetration weld that did *not propagate through the weld nor into the surrounding base metal, and could be detected only by removal of the weld backing or through the use of nondestructive testing (NDT). These conditions were termed types W1a, W1b, and W5.*

As defined in FEMA-267, type W5 consisted of small discontinuities at the root of the weld, which, if discovered as part of a construction quality control program for new construction, would not be rejectable under the AWS D1.1 provisions. FEMA-267 recognized that W5 conditions were likely to be the result of acceptable flaws introduced during the initial building construction, but included this classification so that such conditions could be reported in the event they were detected in the course of the ultrasonic testing (UT) that FEMA-267 required. There was no requirement to repair such conditions. Since these Recommended Criteria do not require UT as a routine part of the inspection protocol, W5 conditions are unlikely to be detected and have been omitted as a damage classification.

Type W1a and W1b conditions, as contained in FEMA-267, consisted of discontinuities, defects and cracks at the root of the weld that would be rejectable under the AWS D1.1 provisions. W1a and W1b were distinguished from each other only by the size of the condition. Neither condition could be detected by visual inspection unless weld backing was removed, which, in the case of W1a conditions, would also result in removal of the original flaw or defect. At the time FEMA-267 was published, there was considerable controversy as to whether or not the various types of W1 conditions were actually damage or just previously undetected flaws introduced during the original construction. Research conducted since publication of FEMA-267 strongly supports the position that most, if not all W1 conditions are pre-existing defects, rather than earthquake damage. This research also demonstrated that W1 conditions are difficult to detect reliably unless the weld backing is removed. In a number of case studies, it has been demonstrated that when W1 conditions are indicated by UT, they are often found not to exist when weld backing is removed. Similarly, in other cases, upon removal of backing, W1 conditions were found to exist where none had been detected by UT. For these reasons, in the development of these recommendations, it has been decided to de-classify W1 conditions as damage and to eliminate the need for routine use of UT in the performance of detailed connection inspections.

Notwithstanding the above, it is important to recognize that a very significant amount of the "damage" reported following the Northridge earthquake was type W1 conditions. Studies of 209 buildings in the city of Los Angeles have shown that approximately 2/3 of all reported "damage" conditions were type W1. Although these Recommended Criteria do not classify W1 conditions as damage, their presence in a connection can lead to a significant increase in the vulnerability of the building to earthquake induced connection fracture. If, in the performance of connection inspections or repairs it is determined that rejectable *discontinuities, lack of fusion, slag inclusions or cracks exist at the root of a weld, they should be reported and consideration should be given to their repair, as a correction of an undesirable, pre-existing condition.*

Type W2 fractures extend completely through the thickness of the weld metal and can be detected by either magnetic particle testing (MT) or visual inspection (VI) techniques. Type W3 and W4 fractures occur at the zone of fusion between the weld filler metal and base material of the girder and column flanges, respectively. All three types of damage result in a loss of tensile capacity of the girder flange to column flange joint and should be repaired.

2.2.4 Shear Tab Damage

Six types of damage to girder-web-to-column-flange shear tabs are defined in Table 2-4 and illustrated in Figure 2-5. Severe damage to shear tabs is unlikely to occur unless other damage has also occurred to the connection, i.e., column, girder, panel zone, or weld damage, as previously defined.

Table 2-4 Types of Shear Tab Damage

Type	Description
S1	Partial crack at weld to column
S2	Fracture of supplemental weld
S3	Fracture through tab at bolts or severe distortion
S4	Yielding or buckling of tab
S5	Loose, damaged or missing bolts
S6	Full length fracture of weld to column

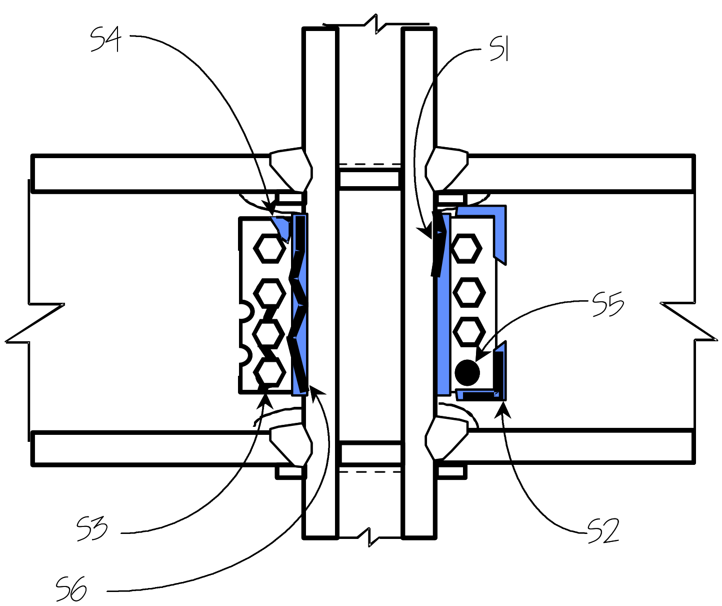

Figure 2-5 Types of Shear Tab Damage

Commentary: Shear tab damage should always be considered significant, as failure of a shear tab connection can lead to loss of gravity-load-carrying capacity for the girder, and potentially partial collapse of the supported floor. Severe shear tab damage typically does not occur unless other significant damage has occurred at the connection. If the girder flange joints and adjacent base metal are sound, they prevent significant differential rotations from occurring between the column and girder. This protects the shear tab from damage, unless excessively large shear demands are experienced. If these excessive shear demands do occur, then failure of the shear tab is likely to trigger distress in the welded joints of the girder flanges.

2.2.5 Panel Zone Damage

Nine types of damage to the column web panel zone and adjacent elements are defined in Table 2-5 and illustrated in Figure 2-6. This class of damage can be among the most difficult to detect since elements of the panel zone may be obscured by beams framing into the weak axis of the column. In addition, the difficult access to the column panel zone and the difficulty of removing sections of the column for repair, without jeopardizing gravity load support, make this damage among the most costly to repair.

Table 2-5 Types of Panel Zone Damage

Type	Description
P1	Fracture, buckle or yield of continuity plate
P2	Fracture in continuity plate welds
P3	Yielding or ductile deformation of web
P4	Fracture of doubler plate welds
P5	Partial depth fracture in doubler plate
P6	Partial depth fracture in web
P7	Full or near full depth fracture in web or doubler
P8	Web buckling
P9	Severed column

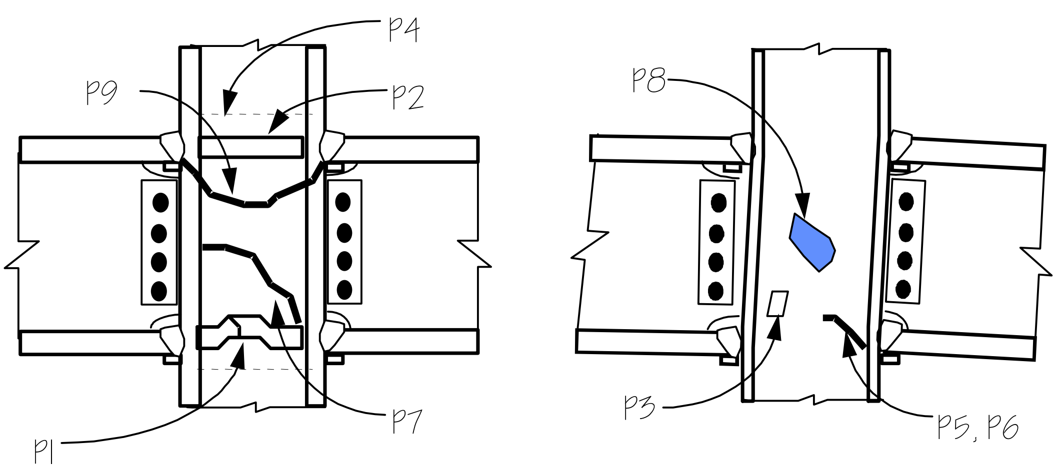

Figure 2-6 Types of Panel Zone Damage

Commentary: Fractures in the welds of continuity plates to columns (type P2), or damage consisting of fracturing, yielding, or buckling of the continuity plates themselves (type P1) may be of relatively little consequence to the structure, so long as the fracture does not extend into the column material itself. Fracture of doubler plate welds (type P4) is more significant in that this results in a loss of effectiveness of the doubler plate and the fractures may propagate into the column material.

Although shear yielding of the panel zone (type P3) is not by itself undesirable, under large deformations such shear yielding can result in kinking of the column flanges and can induce large secondary stresses in the girder-flange-to-column-flange connection.

Fractures extending into the column web panel zone (types P5, P6 and P7) have the potential, under additional loading, to grow and become type P9 (a complete disconnection of the upper half) of the column within the panel zone from the lower half, and are therefore potentially as severe as column splice failures. When such damage has occurred, the column has lost all tensile capacity and its ability to transfer shear is severely limited. Such damage results in a total loss of reliable seismic capacity.

Panel zone web buckling (type P8) may result in rapid loss of shear stiffness of the panel zone with potential total loss of reliable seismic capacity. Such buckling is unlikely to occur in connections that are stiffened by the presence of a vertical shear tab for support of a beam framing into the column's minor axis.

2.2.6 Other Damage

In addition to the types of damage discussed in the previous sections, other types of structural damage may also be found in steel moment-frame buildings. Other framing elements that may experience damage include: (1) column base plates, beams, columns, and their connections that were not considered in the original design to participate in lateral force resistance, and (2) floor and roof diaphragms. In addition, large permanent interstory drifts may develop in structures. Based on observations of structures affected by the 1994 Northridge earthquake, such damage is unlikely unless extensive damage has also occurred to the lateral-force-resisting system. When such damage is discovered in a building, it should be reported and repaired, as suggested by later sections of these *Recommended Criteria*.